COMMERCIAL IN CONFIDENCE

202/79
PR18: Y5/DEY/2
WESC(79) P420

WAVE ENERGY STUDY

NEL OSCILLATING WATER COLUMN (WAVE PISTON)

BOTTOM STANDING DEVICE

UPDATED INTERIM STUDY

for

Department of Energy

Report No. PR18: Y5/DEY/2 November 1979

NATIONAL ENGINEERING LABORATORY

Department of Industry

This report is issued as part of the contract under which the work has been carried out for the sponsor. Any restrictions in the contract on the release of information to third parties apply to this report.

Department of Industry
NATIONAL ENGINEERING LABORATORY
East Kilbride Glasgow Scotland
Telex 77588 Telephone 0355 2 (East Kilbride) 20222

WAVE ENERGY STUDY
NEL OSCILLATING WATER COLUMN (WAVE PISTON)
BOTTOM STANDING DEVICE
UPDATED INTERIM STUDY
NOVEMBER 1979

SUMMARY

This report updates the original report PR7 dated June 1979 and its addendum dated September, 1979 on the feasibility of a Bottom Standing Oscillating Water Column Wave Energy Device. The information derived from various specialist sub studies is incorporated.

Roxburgh and Partners
Mirren House
6 Maxwell Street
Paisley

National Engineering Laboratory
East Kilbride
Glasgow

CONTENTS Page No.

1. INTRODUCTION

During the development of the second Interim Reference Design for the National Engineering Laboratory Oscillating Water Column Wave Energy Device (reference 1) it became apparent that there would be considerable problems associated with the moorings and power transmission riser.

It was seen that these areas represented approximately 21% of the capital cost and 18% of the continuing maintenance cost of the floating device. These costs could be avoided by a bottom standing structure.

Accordingly this feasibility study was initiated by NEL to establish the basic concept for a bottom standing oscillating water column. It was assumed that the device would be situated off the west coast of the Outer Hebrides.

On the basis of the study carried out for the floating units two principal design criteria were identified.

1. As little construction work as possible should be performed at the offshore site location.

2. The amount of structure required to resist long term hydrostatic pressures should be minimised.

2. GENERAL DESCRIPTION

The NEL oscillating water column device extracts energy from the waves as follows:-

1. Primary System

   A column of water is induced to oscillate by the action of the waves. Its motion is used to pump air through the secondary system.

2. Secondary Conversion System

   a. The oscillating air flow is rectified by a louvre type valve to uni directional flow.
   b. This air flow drives a reaction turbine which in turn drives an electric alternator.
   c. The alternating current produced is collected and transmitted to shore.

For the purposes of studying a bottom standing device four water columns with associated equipment have been chosen to form one unit, with several units electrically linked to form a power station. Each unit is constructed in concrete with overall dimensions of 80m long 44 m wide and 32 m high.

Construction of the concrete structure is carried out partly in a dry basin and partly at a sheltered inshore construction berth. The unit is then towed out to its operating site for installation.

At the offshore operating site the device is ballasted down on to a prepared foundation and secured against the fifty year storm by use of rock anchors. Mechanical and electrical equipment which is in the main contained in removable modules is then installed and connected up.

3. SITE CONDITIONS

A preliminary examination of possible sites off the Outer Hebrides was carried out by Rendel Palmer & Tritton (reference 2). Another study by them covering the suitability of sites on the west and north coasts of Scotland and the Islands has confirmed the findings of the earlier report (reference 3). The most suitable sites at the 20m water depth occurred at a distance of 2 - 3 km offshore.

Rendel Palmer & Tritton stated that published geological maps indicated that the bed rock at the chosen site and over quite large areas to the north and south would probably be Lewisian gneiss with the possibility of intrusions of granitic type rocks. They found that the surface of the sea bed was rocky and undulating with loose boulders and pockets of sand and other sediments. A report prepared for NEL by the Scottish Marine Biological Association, Dunstaffnage Laboratory (reference 4) confirms the above finding. The report also covers the likely fouling of the structure by Marine plants and animals.

The most significant plant found at the site is a Laminaria Hyperborea (kelp). It consists of a very tough stalk known at the stipe which terminates in a leaf like growth called the frond. The stipe grows over a period of 6 - 7 years to lengths of 2 metres and diameters of 80mm at the site depth. The fronds also grow to similar lengths to the stipes. They are cast annually and thereafter decompose rapidly to very fine suspended particulate matter.

The stipes attach to solid horizontal or gently inclined substrates using powerful sucker attachments known as holdfasts. During its early years the plant generally grows new holdfasts annually which ensures that it maintains its grip on the rocks. However as it gets older the holdfast growth is reduced and eventually the plant becomes detached. It then drifts along the sea bed until it either decomposes or is cast ashore. It can take up to six months for the stipe to decompose completely.

The maximum size to which the kelp grows is severly limited by the amount of light available. Its population and size falls off rapidly after the 25m depth. Its growth is also inhibited by the deposit of drifting sand or silt which prevents completion of the fertilisation procedure.

There are a number of plant species present at the site but with the exception of the large sponge halichondria panicea none are expected to cause significant fouling. The sponge although it could reach lengths up to 1½m is soft and is easily detachable. The report from the SMBA confirms that it is unlikely that any of the existing plants or animals found in the area would cause significant problems of fouling to a bottom standing OWC structure.

Lewisian gneiss is a massive metamorphic rock of pre Cambrian period formed mainly from feldspar and quartz with a banded texture called foliation. This latter characteristic makes the gneiss susceptible to weathering although this is not expected to be a problem at the Hebridean site once below the surface of the rock. Sound gneiss can allow very high foundation bearing pressures. Typical, physical and mechanical properties are given in table 1 (from Chapter 9 of reference 5).

4. FOUNDATIONS

Prior to the installation of a multi unit power station the site would be surveyed and the foundations method selected to suit the local sea bed conditions.

Several possible foundations methods have been identified. These are as follows:-

1. A Levelled Sea Bed

The sea bed is initially cleared of sea weed and loose material. A sloping or very uneven sea bed is levelled by dredging and blasting if necessary. Suitable plant for such an operation has been identified as the Stevin 80 type of semi-submersible jack up dredging platform. This machine has sufficient power to operate either a hard soil cutter or if necessary a mining road header type cutter in order to level off previously blanket blasted rock head. The machine is reputed to be able to cut a trench up to 80 m wide to a level tolerance of +/- 10cm and it could therefore be expected that no blinding layer of material would be required on a suitable rock foundation.

The structure would be ballasted down using its own flooding valves and that of the special catamaran type installation barge. The unit would then be secured to the foundation using rock anchors drilled in through prepared holes in the structure.

The cavities beneath the foundation of the structure would ultimately be grouted up with a prepared cement grout.

As an alternative with a more uneven rock head the levelling by blasting might be supplemented with large rock infill to create a level bed for installation of the structure. This material would ultimately be grouted after the structure had been installed.

A report on the marine operations and installation of the structures has been prepared for NEL by London Offshore Consultants Ltd (reference 6) is very dubious of the stability of the rock infill in such a situation and care would require to be exercised in choosing the size of the material in order to avoid its dispersion by the force from the waves at the site.

Similar methods have been used to provide foundations for offshore lighthouses in Sweden (reference 7) and a tidal power plant in the USSR (reference 8).

2. <u>Pile Foundations</u>

Hollow circular piles are drilled a short distance into sound rock. The hole is extended below the toe of the pile and a concrete plug with integral ducts placed to secure the pile. The piles are then cut off at a designed level just above the sea bed. The unit, with its underside prepared to mate with the piles, is floated over and ballasted down on to the piles. Rock anchors are then placed through the ducts and stressed to secure the structure.

Similar methods have been used in foundations for bridges in Denmark and Japan (referneces 9 and 10).

3. Pier Foundations

This method is similar in principle to Method 2, but instead of a large number of piles a small number of piers are used to provide a level base on which to place the structure. Caissons, shaped to fit the sea bed contours, are set on the sea bed and filled to the designed level with concrete. The unit is ballasted on to the piers and any irregularities filled with grout. Rock anchors are then drilled through the pier to secure the structure.

Method 1 is preferred by the nominated consultant as it is considered that the preparation of the sea bed would be less costly than the structural work entailed in achieving the necessary dimensional accuracy at the interface between the piles or piers and the structure.

Accordingly the rest of this report assumes that method 1 has been adopted.

5. STRUCTURAL CONSIDERATIONS

A possible structural configuration is shown on drawing No. 1678/71/3RD/1. This arrangement which is an evolution of the second interim reference design reduces the amount of hydrostatic resisting structure by situating the onboard equipment entirely above mean sea level and flooding the ballast areas at the rear of the structure. A wide base is provided to assist in reducing the draft while the structure is floating during the construction stages and to reduce the bearing pressure under the foundation after installation.

The size of the structure shown on the drawing is restricted to 80m long x 44m wide to minimise the problems associated with providing an acceptable foundation.

With the location of the mechanical equipment entirely above sea level and the flooding of the ballast compartments there is very little buoyancy in the structure. Sufficient buoyancy during construction and installation is provided by a combination of temporary bulkheads across the mouth of the water columns and at final installation by the assistance of a catamaran type installation barge.

The mechanical and electrical equipment has now been modularised. There are four removable modules which house respectively the turbo alternator including the spiral casing, the control gear and the two pairs of inlet/outlet rectifier valves. The ducting for the most part is now a permanent part of the structure.

The rock anchors required to secure the structure are situated mainly in the walls surrounding the water column chamber. The preliminary stability calculations show that 152 vertical and 36 inclined rock anchors are required for the 80m long structure shown. Each anchor provides a force of 3610 kN using a 19/18 Dyform prestressing cable.

Current design trends in rock anchor technology are leading to anchor capacities in the order of 10 - 20 MN (see reference 11). However anchors of this size require high strength tendons which are very bulky and require considerable structural space. There fore medium capacity anchors were chosen for ease of handling and installation on this structure.

Prior to the installation of the mechanical and electrical plant the areas of the structure upon which the modules will be placed will form suitable working platforms for the rapid installation of the rock anchors.

The larger number of medium capacity anchors also help to distribute the compressive prestress effect through the structure. The vertical anchors in the walls provide a direct compressive stress which reduces crack widths occurring under serviceability loadings. This allows the amount of ordinary reinforcement to be reduced.

Corrosion protection to the anchors is provided by a double system which consists of a cement grout surrounding either a high strength epoxy or polyester resin coating in the fixed length or a water resistant grease packed plastic sheath in the free length. The anchorages in the structure will be covered with filled caps and enclosed in epoxy resin blocks.

The cost and effectiveness of the rock anchors depends entirely on the soundness of the gneiss underlying the structure. At present there is no detailed information available about the geology of the selected site. A full geological site investigation will be required to enable further detailed design work to be carried out.

The NEL has commissioned a sub study by Colcrete Ltd on the installation and use of rock anchors in the structure under consideration. The conclusions of this report are that the work is entirely possible (reference 12).

This study has concentrated on establishing the minimum overall structural dimensions for a water column with fundamental dimensions similar to those used in the second interim reference design. Further work is required to quantify how these dimensions are affected by the proximity of the sea bed and by the fixity of the structure.

6. CONSTRUCTION & INSTALLATION

It is proposed that the units are built in a similar manner to concrete North Sea oil production platforms. With dimensions shown on drawing No. 1678/71/3RD/1 it would be possible to construct several units at the same time in one basin or to make use of one of the large shipbuilding or repair dry docks.

A construction sequence will be as follows:-

1. The base slab is first constructed in the dry basin.
2. The main vertical walls are then constructed using fixed shuttering and slipforming as appropriate to a height sufficient to allow float out from the basin.
3. Temporary bulkheads are then installed across the mouth of the water columns. The basin is flooded and the structure is floated out.
4. At a sheltered inshore berth the walls are completed and precast floor slabs are placed in position in the bottom of the water column chamber. The roof slabs to the chamber are then constructed.
5. The permanent parts of the mechanical installation such as the ducting are installed.
6. The unit is then ready to tow to the offshore installation site.

Preparation of the foundations at the offshore site is carried out concurrently with the construction of the structure.

The preparation sequence is as follows:-

1. Sea weed, sand, boulders and other loose material and weathered rock is removed from the foundation areaby dredging, scraping and high powered water jets.

2. If necessary the area is levelled by drilling and blasting. The area is then cut down to a level by removing the shattered rock with a rock cutter head on the dredging plant.

(Note: A suitable piece of plant is currently under construction for Stevin Dredging in Holland. This is the Stevin 80 which is a large semi submersible cutter dredger capable of operating in offshore conditions. The dredger is fully self propelled when floating and is classified 100 A 1 for riding out any storm which might be encountered. In the operating mode the plant can perform in significant waves up to 2.5m height and the dredge walk forward on its operating legs at a speed of 20m per hour. In the context of a 2 GW installation of wave energy structures numbering some 1500 + the suggestion of dedicating the £50 m piece of plant such as this seem perfectly logical).

3. The foundation for the structure should be capable of being prepared to a tolerance of +/- 10cm. using the dedicated dredging plant. Should any low areas require make up of broken rock it is possible that this could be done by careful choice of the size of material. The report commissioned by NEL on the marine operations and installation of the structures suggests that a total under keel clearance of 5 m will be required to ensure the safe towage and installation on the site. In order to provide this, extra buoyancy will be required for installing the structures on site and this has been allowed for as follows:

1. The structure arrives under tow from deep water at a sheltered location near installation site.

2. The structure is taken over at the sheltered water holding site by tugs and a dedicated installation barge. This structure is a catamaran type barge with several portal type structures joining the two hulls and a working deck containing ballast tanks enclosing the portals.

3. The bottom standing device is placed between the catamaran hulls and the portal frames attached to the top of the structure.

4. The catamaran hulls are jacked down the portal frames and provided additional buoyancy to reduce the draft of the bottom standing structure to give the 5 m under clearance required for towing to the installation site. The structure is towed to site and ballasted down on to its prepared foundation.

5. The catamaran hulls are now completely filled with ballast water and then jacked up their supporting framework to provide additional ballast to maintain the structure firmly on its foundation during the installation of the rock anchors. Ballast water is also introduced into the working deck tanks to ensure the stability of the structure.

6. The rock anchors are installed, tensioned and grouted and when sufficient of these have been provided the installation barge is deballasted and removed for installation of subsequent structures.

7. Any necessary grouting is carried out underneath the base of the structure to secure it permanently to the sea bed.

8. The bulkhead doors are removed and taken back to the construction yard for use with further structures.

For the particular configuration prepared for this report there was a significant constraint to the installation procedure at stages 4 to 6. When the structure was initially ballasted down on to its foundation and prior to the installation of the rock anchors the maximum wave height which could be resisted was approximately 3m. With the provision of a dedicated catamaran type installation barge which could be jacked up on to the structure to provide extra ballast weight the height of wave which could be resisted without any rock anchor in place was increased to 14m. This is considered satisfactory to allow the safe installation of all the rock anchors.

7. MAINTENANCE

A bottom standing OWC device situated at the Outer Hebridean site may be subject to loss of efficiency caused by accumulation of sand and organic debris at the entrance, floor and rear bottom corner of the water column chamber, and by encrustation by barnacles, mussels and growth of sponges on the interior walls. However, it is understood that due to the lack of light and the presence of silt and sand the growth of the principal seaweed in the location i.e. the kelp will be practically non existent inside the chamber.

It is not possible at this time to predict the extent of the fouling and sedimentation. However its removal would be incorporated into the periodic maintenance schedule.

An important feature of the proposed structure is that the entrance to the water column chamber may be closed off using steel stop logs. These would be located in guides on the front wall of the structure. It would then be possible to dewater the chamber thereby giving 'dry' access for cleaning and inspection.

Research into the ecosystem of the area is presently being carried out by the Dunstaffnage Marine Research Laboratory. A report has been prepared for NEL by Staff of the Laboratory (reference 4). The report indicates that there should be no great fouling of the structure and its water column and that the removal of such fouling plants and animals should be a simple matter during routine maintenance.

8. WAVE PARAMETERS

This report makes a preliminary investigation into the stability of the structure and the pressures under the foundation. Accordingly it is considered that the worst cases will be satisfactorily given by adopting an equivalent maximum static design wave appraoch. In accordance with the Department of Environment Guidance Notes (reference 13) this would normally be taken as the wave having an average recurrence period of 50 years.

However, at the selected site the maximum design wave parameters are considerably modified by the proximity of the sea bed. The most important effects are shoaling, breaking and refraction. The breaking effect controls the maximum wave height because when the depth decreases to the same order of magnitude as the wave height, waves that are within a certain range of steepness (i.e. height/length) become unstable and break. A CIRIA report (reference 14) gives the following values for a mean water depth of 16 - 20m.

Water period	Maximum Wave Height
4 - 10 secs	4 - 14 m
above 10 secs	15 m

A more detailed study indicates that the above figures are reasonable at 20m depth.

9. WAVE LOADINGS

In order to arrive at a method for calculating horizontal wave loadings, it has been assumed that the OWC device acts like a solid vertical breakwater.

This assumption will give conservative results because the water column mechanism will either absorb or attenuate some of the wave energy thereby reducing its peak effect.

Horizontal wave forces can arise from two types of wave. The first is the oscillatory wave which becomes reflected and thereby sets up a standing wave known as a 'clapotis'. The height of the clapotis is generally greater than the height of the approaching wave resulting in maximum overall forces on the structure. The second type of wave is the translatory (i.e. breaking) wave which is more likely to cause critical forces on local areas of the structure.

The force on the structure caused by the clapotis is generally calculated using formulae which take account of the dynamic action of the water particles colliding with the face of the structure. The currently most acceptable theory is that due to Miche which takes into account 2nd order effects (from reference 15).

Using a maximum wave height of 15m at a period of 13 secs, the Miche gives a horizontal wave force of 413 tonnes per m length of structure acting at a distance of approximately 13 m above the sea bed when the depth of water is 20 m.

10. STABILITY CALCULATIONS

Preliminary calculations for stability against sliding show that the structure cannot mobilise sufficient frictional resistance under the base without the extra downwards force provided by rock anchors. With sufficient anchors installed to provide adequate horizontal resistance the overturning stability is then very high.

It was determined from Terzaghi and Peck (reference 16) that the coefficient of friction between the base of the structure and the underlying rock could be as high as 0.45. This value is not reduced by submergence as long as the normal force is calculated on the overall weight of the structure less the uplift due to the displaced volume of those parts that are immersed.

The calculation for sliding stability is presented in Table 2. This shows that a minimum factor of safety of 1.5 may be obtained using 36 inclined anchors and 152 vertical anchors.

11. COSTS

Comparitive output figures for the bottom standing and floating OWC devices have been presented in Tables 3.1 & 3.2 along with those given for the HRS device and an optimistic best device (from reference 17).

The cost/unit has been calculated based on the figures contained in Tables 3 and using the best estimates to date of the cost of the structures.

A range of unit costs are given for both upper and lower bound 95% confidence limit values of the power delivered.

Current studies of the structures and installation cost indicate some reductions are likely. However, recent output figures derived from the latest studies of the wave energy available tend to indicate a lower average power production. Both these studies are at too early a stage to warrant inclusion in the tables of this report.

12 FURTHER WORK

The estimates given in Table 4.2 indicate that the Bottom Standing Structure could produce power and cost/unit which approaches the target of 10p/kWh. However further work is required in several areas to ensure that costs are reduced to a minimum.

1. The underwater geology of the Hebridean coastline requires detailed investigation of the gneiss bedrock particularly with respect to its extent and soundness.

2. The installation method for the structure requires further study and refinement to enable more precise costings to be made.

3. The structure requires more detailed analysis to ensure that the most economic configuration is being presented.

4. The study must be extended to cover sites without bedrock.

5. Further collection and analysis of wave data are required to give better estimates of forces on the structure and the likely power outputs.

REFERENCES

1. NATIONAL ENGINEERING LABORATORY

   Wave Energy Study, NEL Oscillating Water Column, 100 MW Power Station, 2nd Interim Reference Design. Report No. PR2 Y5/DEY/2. July 1978

2. RENDEL, PALMER & TRITTON in association with KENNEDY & DONKIN

   United Kingdom Wave Energy Programme, Consultants report on sites for Sea bed mounted devices in Outer Hebrides June 1978.

3. RENDEL, PALMER & TRITTON in association with KENNEDY & DONKIN

   Assessment of Total Potential Power Production from the West and North Coasts of Scotland using Sea-bed Mounted Devices. September 1979

4. SCOTTISH MARINE BIOLOGICAL ASSOCIATION

   Report on the Environmental Factors Relating to Sites for the Bottom Standing Structures West of the Outer Hebrides. M.J. Picken - September 1979

5. BLAKE L.S. (Editor)

   Civil Engineer's Reference Book
   Newnes - Butterworths, 3rd Edition 1975.

6. LONDON OFFSHORE CONSULTANTS LTD

   Report into the Feasibility of installing the NEL Bottom Standing Wave Energy Device - October 1979.

7. ERIKSSON K.

   Concrete caisson for offshore lighthouses.
   Proc. of the FIP Symposium on Concrete Sea Structures. Tbilisi, September, 1972.

REFERENCES (contd)

8. BERNSTEIN L.B.

Kislogubskaya tidal power plant.
Proc. of the FIP Symposium on Concrete Sea structures.
Tbilisi, September 1972.

9. OSTENFELD CHR.

Caissons for bridge piers and the new Lillebaelt Bridge - design and construction of piers.
Proc. of the FIP Symposium of Concrete Sea Structures. Tbilisi, September 1972.

10. SHIRAISHI S.

Construction of large concrete foundations in the sea. Proc. of the FIP Symposium on Concrete Sea Structures. Tbilisi, September, 1972.

11. LITTLEJOHN G.S. & BRUCE D.A.

Rock anchors - start of the art.
Foundation Publications 1977.
(Originally published in 'Ground Engineering' magazine between May 1975 and May 1976.)

12. COLCRETE LTD

NEL Oscillating Water Column, Bottom Standing Device - Preliminary Comments on Rock Anchors - Dr. D.A. Bruce - October 1979

13. DEPARTMENT OF ENERGY

Offshore installations: Guidance on design and construction 2nd Edition, 1977.

REFERENCES (contd)

14. CIRIA UNDERWATER ENGINEERING GROUP

Dynamics of Marine structures: Methods of calculating the dynamic response of fixed structures subject to wave and current action. Report no. UR8. 1977.

15. WIEGEL, R.L.

Oceanographical Engineering
Prentice Hall International1st Edition, 1964.

16. TERZAGHI K & PECK R.B

Soil Mechanics in Engineering Practice.
John Wiley & Sons, 2nd Edition, 1967.

17. RENDEL, PALMER & TRITTON in association with KENNEDY & DONKIN

United Kingdom Wave Energy Programme, Notes for the Consultants' Presentation, Wavepower Workshop at Heathrow. November, 1978.

TABLE 1

Typical Physical and Mechanical Propertiesx of Gneiss

Porosity	1%
Dry bulk density	26 to 28 kN/m^3
Uniaxial compressive strength	200 MN/m^2
Uniaxial tensile strength	30 MN/m^2
Young's modulus (laboratory value)	50×10^3 MN/m^2
Poissons ratio (laboratory value)	0.1
Presumed bearing value (from CP 2004)	10 MN/m^2

TABLE 2

Values of Lateral Forces

		t/m
Horizontal wave force		413
Total structure weight	515	
less displacement	162	
Net vertical force	353	
Frictional resistance mobilised (angle of friction = 24^{o})		159
Extra resistance to be provided by anchors		254

No. of inclined anchors = 36

No. of vertical anchors = 152

Horizontal force provided by inclined anchors		83
Vertical force due to inclined anchors	144	
due to vertical anchors	702	
	846	
Frictional resistance that can be mobilised (angle of friction = 24^{o})		381
Extra resistance provided by anchors		464

$$\text{o/a factor of safety} = \frac{159 + 83 + 381}{413} = 1.51$$

Table 3.1   Output Predictions for 78 Reference Designs   Figures for Devices 1 - 3 from RPT presentation notes for Heathrow Wavepower Workshop, November 1978.

	KEY: (HIGH ESTIMATE) MOST LIKELY (LOW ESTIMATE)	Annual Apparent Power at S Uist Buoy	Shallow Water Correction (As Captured)	Site Correction (Energy Loss and Shielding)	Direction-ality Correction	Device Capture Efficiency		Power Chain		Power Delivered to Perth
						Based on PM Spectra	Digital Spectrum Correction	Effici-ency	Reliab-ility	UPPER BOUND 95% CONFIDENCE LIMIT MEAN LOWER BOUND 95% CONFIDENCE LIMIT
No.	DEVICE	kW/m	fsw	f site	fd	d	f digital	p	fr	kW/m
1	NEL 78 Reference Design	(46)		(1.15)	(0.75)	(0.44)		(0.55)	(0.92)	(5.7)
		42.3	1.13	1.1	0.65	0.39	0.92	0.37	0.87	4.2
	Floating with Hydraulics	(39)		(1.0)	(0.50)	(0.34)		(0.33)	(0.80)	(3.1)
2	HRS	(46)		(1.0)	(0.75)	(0.45)		(0.60)	(0.95)	(4.9)
		42.3	1.13	0.9	0.65	0.33	0.92	0.41	0.92	3.2
		(39)		(0.7)	(0.50)	(0.21)		(0.35)	(0.83)	(1.9)
3	OPTIMISTIC BEST DEVICE Scenario 2	42.3	1.13	1.1	0.7	0.6	0.92	0.7	0.95	13.5

Table 3.2

Output Comparisons for NEL Devices

Figures prepared with advice and assistance from ETSU and RPT

	Key: COLS 1 - 8 (HIGH ESTIMATE) MOST LIKELY (LOW ESTIMATE) [AVERAGE VALUE]	(1) Annual Apparent Power At S Uist Buoy	(2) Shallow Water Correction (As Captured)	(3) Site Correction (Energy Loss and Shielding)	(4) Direction-ality Correction	(5) Device Capture Efficiency Based on PM Spectra	(6) Digital Spectrum Correction	(7) Power Chain Effic-iency	(8) Reliab-ility	(9) Power Delivered To Perth UPPER BOUND 95% CONFIDENCE LIMIT MEAN LOWER BOUND 95% CONFIDENCE LIMIT
No.	DEVICE	kW/m	fsw	f site	fd	d	f digital	p	fr	kW/m
1A	NEL Floating 78 Reference Design RPT Report with Hydraulics	(46.0) 42.3 (39.0)	1.13 [1.13]	(1.15) 1.1 (1.0) [1.083]	0.78 [0.78]	(0.44) 0.39 (0.34) [0.39]	0.92 [0.92]	(0.55) 0.37 (0.33) [0.417]	(0.92) 0.87 (0.80) [0.863]	(6.8) 5.2 (4.0)
4A	NEL Bottom Standing Device Team No Hydraulics	(46.0) 42.3 (39.0)	1.13 [1.13]	(0.75) 0.61 (0.45) [0.603]	0.78 [0.78]	(0.82) 0.73 (0.64) [0.73]	0.92 [0.92]	(0.78) 0.61 (0.55) [0.65]	(0.95) 0.92 (0.83) [0.90]	(11.5) 8.8 (6.5)

Also see notes on Table 3.3

Table 3.3

Notes to Table 3.2

1. Directionality correction factor (Column 4) amended to ETSU value as given at NEL Technical Review (TR) Meeting No.2 on 31st July 1979 for devices facing in optimum direction.

2. Site correction factor for bottom standing devices taken as agreed at NEL TR Meeting No.2:

(High Estimate)	-	0.65 x	value for floating device
Most Likely	-	0.55 x	ditto
(Low Estimate)	-	0.45 x	ditto

These figures represent the best available estimate at this time (August 1979).

3 Row 1A - Power chain efficiency taken from RPT Wavepower Workshop Presentation Notes November 78 (as Table 3.1).

Rows 4A - Power chain efficiency taken as that expected from a scheme without hydraulic interconnection of the turbines.

Table 4.1

Estimated Cost of Bottom Standing Device

1. CAPITAL COSTS	Floating	Bottom Standing	per 80m
	£ per m	£ per m	
Body of Structure	86200	62500	£5M
M & E Plant	25800	25000	£2M
Tow and Install	5200	7500	£0.6M
Foundation and rock anchors		12500	£1M
Moorings	34500		
Power take off	20700	5000	£0.4M
Contingencies	17200	12500	£1M
	£189600 /m	£125000 /m	£10M
Take length of device as 40km. NB. Overall length greater.			
Capital Cost of Power Station	£7584 x 10^6	£5000 x 10^6	
Capital Cost of Maintenance Base	£ 100 x 10^6	£ 20 x 10^6	
Total Capital Cost	£7684 x 10^6	£5020 x 10^6	
Annual Maintenance	£ 192 x 10^6	£ 50 x 10^6	
Annual Repayment 25 yrs 5% compound interest (Approx 7.1% simple interest)	£ 546 x 10^6	£ 356 x 10^6	
Total Annual Cost	£ 738 x 10^6	£ 406 x 10^6	

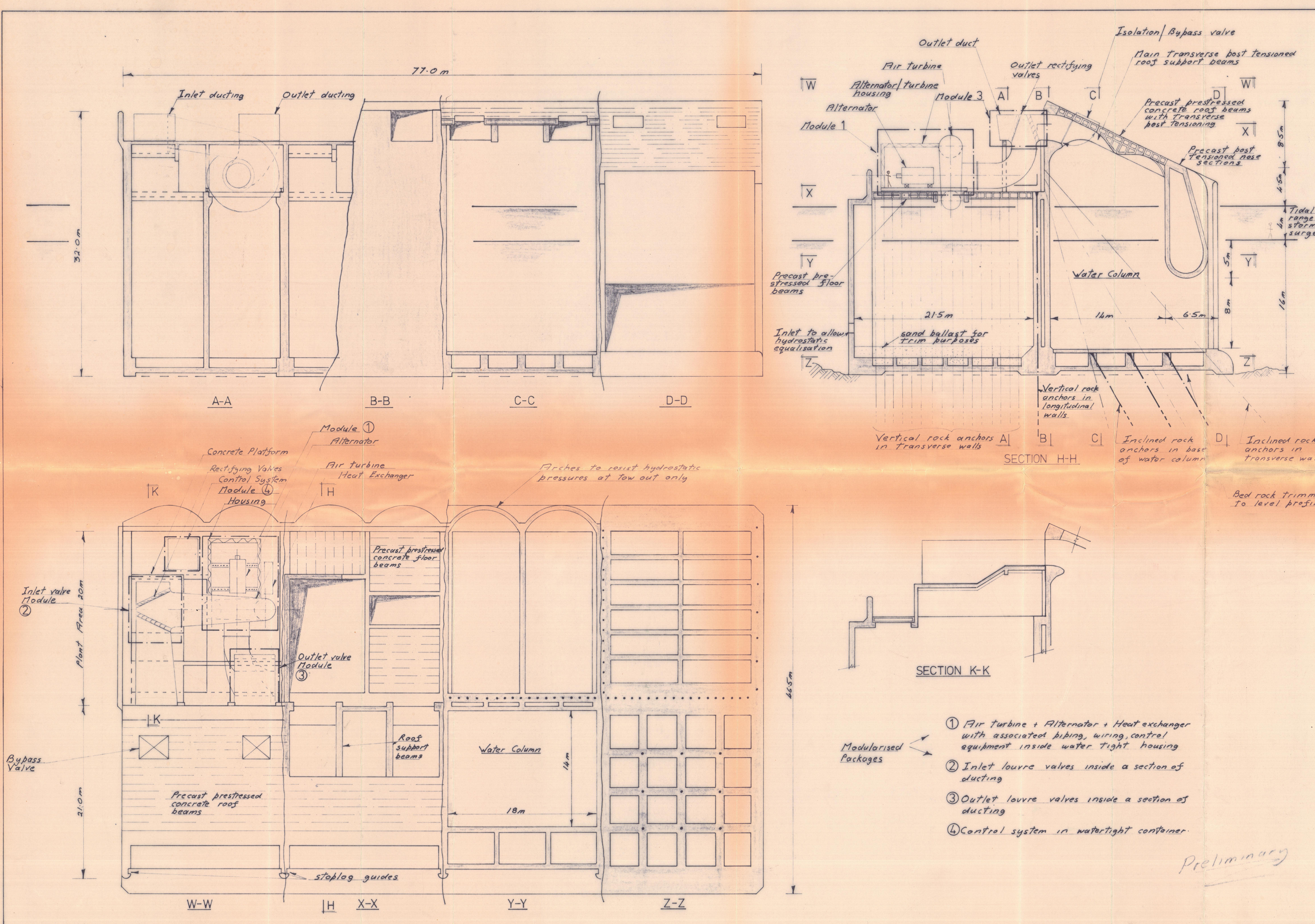
Notes:
1. Cost figures presented are preliminary figures and are intended to be used for comparison between the devices.

Table 4.2

2. POWER DELIVERED	UNITS	UPPER BOUND 95% CONFIDENCE LIMIT VALUES		LOWER BOUND 95% CONFIDENCE LIMIT VALUES	
		Floating	Bottom Standing	Floating	Bottom Standing
Average annual power delivered to Perth per m length of device	kW/m	6.8			
Total annual energy delivered to Perth per m length of device (based on 24 x 365 = 8760hrs)	kWh/m	59568	100740	35040	56940
Length of Device taken as 40km					
Average annual power delivered to Perth per power station	MW	272	460	160	260
Total annual energy delivered to Perth per power station	kWh (units)	2383×10^6	4030×10^6	1402×10^6	2278×10^6
Energy cost in pence per unit	p/kWh	31.0	10.1	52.6	17.8

Notes:

1. Cost figures presented are preliminary figures and are intended to be used for comparison between the devices.



MK.	REVISION	By	Date

NATIONAL ENGINEERING LABORATORY WAVE PISTON

MARCH 1980 REFERENCE DESIGN BREAKWATER TYPE

GENERAL LAYOUT

ROXBURGH & PARTNERS
CONSULTING ENGINEERS
MIRREN HOUSE MAXWELL ST.
PAISLEY SCOTLAND
Tel. 041-889-0044
Telex 779684 ROXBUR G
ABERDEEN LONDON

Job No.	Drawing No.	Revision
1678/71	3RD/1	

Scale	Date
1:200	14/11/79

Drawn By	Passed By	Issued For
ajm	RPR	REPORT PR18